CHAPTER 12

Stable Channel Design Functions

The stable channel design functions are based upon the methods used in the SAM Hydraulic Design Package for Channels, developed by the U.S. Army Corps of Engineers Waterways Experiment Station. This chapter presents the methods and equations used for designing stable channels, including channel geometry, and sediment transport capacity.

Much of the material in this chapter directly references the SAM Hydraulic Design Package for Channels User's Guide (USACE, 1998) and EM 1110-2-1601. There have been a number of alterations to the general approach used in SAM in order to expand its capabilities and to fit within the framework of HEC-RAS. For information on how to enter data for stable channel design and sediment transport capacity analysis, and how to view results, see Chapter 15 of the HEC-RAS user's manual.

Contents

Uniform Flow Computations

For preliminary channel sizing and analysis for a given cross section, a uniform flow editor is available in HEC-RAS. The uniform flow editor solves the steady-state, Manning's equation for uniform flow. The five parameters that make up the Manning's equation are channel depth, width, slope, discharge, and roughness.

$$Q = f(A, R, S, n) \tag{12-1}$$

Where: Q = Discharge
A = Cross sectional area
R = Hydraulic radius
S = Energy slope
n = Manning's n value

When an irregularly shaped cross section is subdivided into a number of subareas, a unique solution for depth can be found. And further, when a regular trapezoidal shaped section is used, a unique solution for the bottom width of the channel can be found if the channel side slopes are provided. The dependant variables A, and R, can then be expressed in the Manning equation in terms of depth, width and side slope as follows:

$$Q = f(Y, W, z, S, n) \tag{12.2}$$

Where: Y = Depth
W = Bottom width
z = Channel side slope

By providing four of the five parameters, HEC-RAS will solve the fifth for a given cross section. When solving for width, some normalization must be applied to a cross section to obtain a unique solution, therefore a trapezoidal or compound trapezoidal section with up to three templates must be used for this situation.

Cross Section Subdivision for Conveyance Calculations

In the uniform flow computations, the HEC-RAS default Conveyance Subdivision Method is used to determine total conveyance. Subareas are broken up by roughness value break points and then each subarea's conveyance is calculated using Manning's equation. Conveyances are then combined for the left overbank, the right overbank, and the main channel and then further summed to obtain the total cross section conveyance. Refer to Chapter 2 for more detail.

Bed Roughness Functions

Because Manning's n values are typically used in HEC-RAS, the uniform flow feature allows for the use of a number of different roughness equations to solve for n. HEC-RAS allows the user to apply any of these equations at any area within a cross section, however, the applicability of each equation should be noted prior to selection. The following bed roughness equations are available:

- Manning Equation
- Keulegan Equation
- Strickler Equation
- Limerinos Equation
- Brownlie Equation
- Soil Conservation Service Equations for Grass Lined Channels

The Manning equation is the basis for the solution of uniform flow in HEC-RAS.

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \qquad (12\text{-}3)$$

Roughness values solved for using other roughness equations are converted to Manning's n values for use in the computations. One n value or a range of n values is prescribed across the cross section and then the Manning's equation is used to solve for the desired parameter.

<u>Manning Equation:</u>

When choosing the Manning equation method, one n value or a range of n values is prescribed across the cross section and then the Manning's equation is used to solve for the desired parameter.

<u>Keulegan Equation:</u>

The Keulegan (1938) equation is applicable for rigid boundary channel design. Flow is classified according to three types: hydraulically smooth, hydraulically rough, or a transitional zone between smooth and rough. To solve the Keulegan equation, a Nikaradse equivalent sand roughness value, k_s must be provided. Values for k_s typically range from $1d_{90}$ for large stones to $3d_{90}$ for sand and gravel with bed forms, where d_{90} is the representative grain size in which 90% of all particles in the bed are smaller. However, k_s values are highly variable and depend also on the types of bed forms, the overall grain distribution, the particle shape factor, and other physical properties. Therefore, unless there is specific data related to the k_s value for a given cross section of a river, it is recommended that one of the other roughness equations be chosen. If the discharge, area, hydraulic radius, and slope are known, a k_s

value can be calculated and then used in the solution of additional discharges, depths, slopes, or widths. EM 1110-2-1601 has a table of suggested k_s values for concrete-lined channels.

Van Rijn (1993) defines the three boundary-zone flow regimes as follows:

Hydraulically smooth flow is defined as flow in which the bed roughness elements are much smaller than the thickness of the viscous sublayer and do not affect the velocity distribution (Figure 12.1) . This is found when

$$\frac{u_* k_s}{\nu} \leq 5 \tag{12-4}$$

Where: u_* = current related bed shear velocity
ν = kinematic viscosity coefficient
k_s = equivalent sand roughness value

Hydraulically rough flow is defined as flow in which a viscous sublayer does not exist and the velocity distribution is not dependent on the viscosity of the fluid (Figure 12.1). This is found when

$$\frac{u_* k_s}{\nu} \geq 70 \tag{12-5}$$

Transitional flow is where the velocity distribution is affected by viscosity as well as by the bottom roughness.

$$5 < \frac{u_* k_s}{\nu} < 70 \tag{12-6}$$

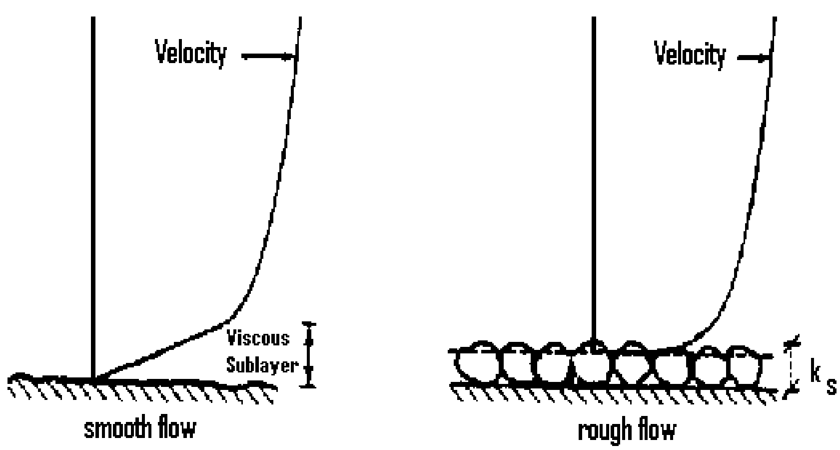

Figure 12.1 Velocity Distribution in Smooth and Rough Flow (Van Rijn, 1993)

The equation for fully rough flow is

$$C = 32.6\log_{10}\left(\frac{12.2R}{k_s}\right) \tag{12-7}$$

Where: C = Chezy roughness coefficient
R = Hydraulic radius

And for fully smooth flow

$$C = 32.6\log_{10}\left(\frac{5.2R_n}{C}\right) \tag{12-8}$$

Where: R_n = Reynolds number

Iwagaki (Chow, 1959) found from experimental data that the coefficients 12.2 and 5.1 actually vary with the Froude number. He reasoned that as the Froude number increases, the stability of the free surface diminishes, creating more resistance in the open channel. According to Iwagaki, for fully rough flow, the coefficient 12.2 should be replaced by

$$10^{\frac{A_r\sqrt{g}}{32.6}} \quad \text{to get} \quad C = 32.6\log_{10}\left[10^{\frac{A_r\sqrt{g}}{32.6}}\left(\frac{R}{k_s}\right)\right] \tag{12-9}$$

Where: A_r = Coefficient for rough flow that varies with Froude number

$$A_r = -27.058\log_{10}(F+9) + 34.289 \tag{12-10}$$

Where: F = Froude number

For fully smooth flow the coefficient 5.2 should be replaced by

$$\frac{\sqrt{g}}{4}10^{\frac{As\sqrt{g}}{32.6}} \quad \text{to get} \quad C = 32.6\log_{10}\left[10^{\frac{As\sqrt{g}}{32.6}}\left(\frac{\sqrt{g}R_n}{4C}\right)\right] \tag{12-11}$$

Where: A_s = Coefficient for smooth flow that varies with Froude number

$$As = -24.739\log_{10}(F+10) + 29.349 \tag{12-12}$$

When the flow is in the transitional regime, the Chezy coefficient is just a combination of the equations for smooth and rough flow.

$$C = -32.6 \log_{10}\left[\frac{k_s}{R10^{\frac{A_r\sqrt{g}}{32.6}}} + \frac{4C}{\sqrt{g}R_n 10^{\frac{As\sqrt{g}}{32.6}}}\right] \qquad (12\text{-}13)$$

It should be noted that the data used to develop these equations had Froude numbers ranging from 0.2 to 8.0. Also, the Keulegan method should not be used when the relative roughness (R/k_s) is less than 3. This indicates extremely rough flow, which does not follow the logarithmic velocity distribution from which Keulegan's method is based. HEC-RAS uses equation 12-13 for uniform flow computations when the Keulegan method is selected. When the flow is fully rough, the relative roughness term of the equation becomes dominant and the viscous effects (R_n) are relatively small. When the flow is fully smooth, the sublayer viscous effects become dominant and the relative roughness term drops out.

Once the Chezy coefficient is determined, it is converted to a Manning's n value for use in the Manning equation from the following expression:

$$n = \frac{1.486}{C} R^{1/6} \qquad \text{(U.S. Customary Units)} \qquad (12\text{-}14)$$

$$n = \frac{1}{C} R^{1/6} \qquad \text{(S.I. Units)}$$

Strickler Equation

When comparing the relative roughness to a so-called Strickler function, it is found that over a wide range of relative roughness, the variation of the Strickler function, $\phi R/k_s$ is small (Chow, 1959). Because of this relationship, a constant value for the Strickler function can be used to calculate an n value. Strickler assumed this constant value to be 0.0342 when k_s and R are given in feet and when the Nikaradse k_s value is given as the d_{50} of the bed sediment. Research at WES (Maynard, 1991) has produced different results when the Strickler function is applied to riprap-lined beds. In this case k_s is the bed sediment d_{90} and the value applied to the Strickler function should depend on the type of calculations when designing channels. For velocity and stone sized calculations, the Strickler function should be 0.0342. For discharge capacity calculations, 0.038 should be used. The following expression converts k_s to an n value.

$$n = \phi \frac{R}{k_s} k_s^{1/6} \qquad (12\text{-}15)$$

Where: k_s = Nikaradse equivalent sand roughness, ft or m, $=d_{50}$ for natural channels and d_{90} for riprap-lined channels.

$\phi R/k_s$ = Strickler function = 0.0342 for natural channels
= 0.0342 for velocity and stone size calculations in riprap design.
= 0.038 for discharge calculations in riprap design.

Limerinos Equation

Larger grain sizes from coarse sands to cobbles were used by Limerinos (1970) to develop an n-value predictor based on Hydraulic roughness and particle sediment size for mobile bed streams. This method can only be applied to the grain-related upper flow regime, which includes plane bed, antidunes, and chutes and pools. Sand bed streams are applicable provided that the bed form is plane bed (Burkham and Dawdy, 1976). Whether a channel is in upper, lower, or the transitional bed form regime is a function of the localized, or Grain-related Froude Number which is defined as the following:

$$F_g = \frac{V}{\sqrt{(s_s - 1)gd_{50}}} \tag{12-16}$$

Where: F_g = Grain-related Froude number
V = Average channel velocity
s_s = Specific Gravity of sediment particles

If the bed slope is greater than 0.006, flow is always considered to be in the upper regime. Otherwise, upper and lower regime can be defined as follows

$$F_g > \frac{1.74}{S^{1/3}} \quad \text{Grain-related upper Regime Flow}$$

$$F_g \le \frac{1.74}{S^{1/3}} \quad \text{Grain-related lower Regime Flow} \tag{12-17}$$

Where: S = Bed Slope

The n-value predictor as defined by Limerinos is:

$$n = \frac{0.0929R^{1/6}}{1.16 + 2.0\log_{10}\left(\frac{R}{d_{84}}\right)} \tag{12-18}$$

Where: R = Hydraulic Radius
d_{84} =the particle size for which 84% of all sediments are smaller

It is important that the Limerinos method be chosen with care. The data ranges at which it applies are relatively small and limited to coarse sands to cobbles in upper regime flow. A particular advantage with the Limerinos method is its apparent accounting for bed form roughness losses. As a consequence, n values computed using Limerinos will normally be significantly higher than those found using Strickler. Burkham and Dawdy showed that the range of relative roughness of the Limerios method is between 600 and 10,000.

Brownlie Equation

Brownlie (1983) developed a method for use with bed forms in both the upper and lower regime. In this method the Strickler function is multiplied by a bed-form roughness, which is a function of the hydraulic radius, the bed slope and the sediment gradation. The resulting equations for lower and upper regime are:

$$n = \left[1.6940 \left(\frac{R}{d_{50}} \right)^{0.1374} S^{0.1112} \sigma^{0.1605} \right] 0.034 \left(d_{50} \right)^{0.167} \quad \text{(Lower Regime)}$$

$$n = \left[1.0213 \left(\frac{R}{d_{50}} \right)^{0.0662} S^{0.0395} \sigma^{0.1282} \right] 0.034 \left(d_{50} \right)^{0.167} \quad \text{(Upper Regime)} \tag{12-19}$$

Where: σ = the geometric standard deviation of the sediment mixture

$$\sigma = 0.5 \left(\frac{d_{84} + d_{50}}{d_{50} + d_{16}} \right) \tag{12-20}$$

In actuality, the transition between the upper and lower regimes does not occur at one point, but rather over a range of hydraulic radii. Within this range, there are actually two valid solutions (a lower and an upper regime solution) because the transition is initiated at different discharges depending on whether the occurrence is on the rising end or falling end of the hydrograph. HEC-RAS will solve for both and when there are two solutions, a message box will appear that requests the user to select which regime to solve for. A general rule of thumb is to use the upper regime for the rising end of the hydrograph and the lower regime for the falling end of the hydrograph (Figure 12.2).

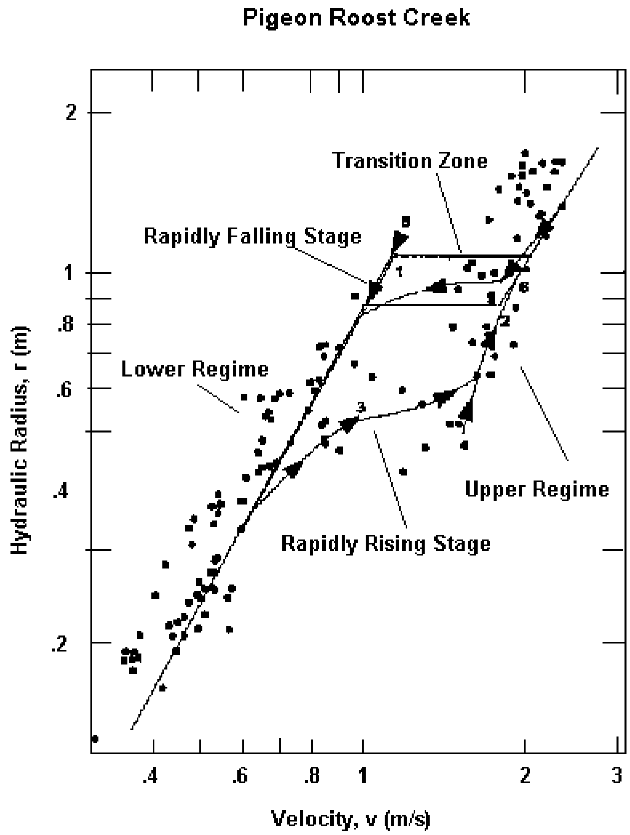

Figure 12.2 Example: Velocity vs. Hydraulic Radius in a Mobile Bed Stream (California Institute of Technology)

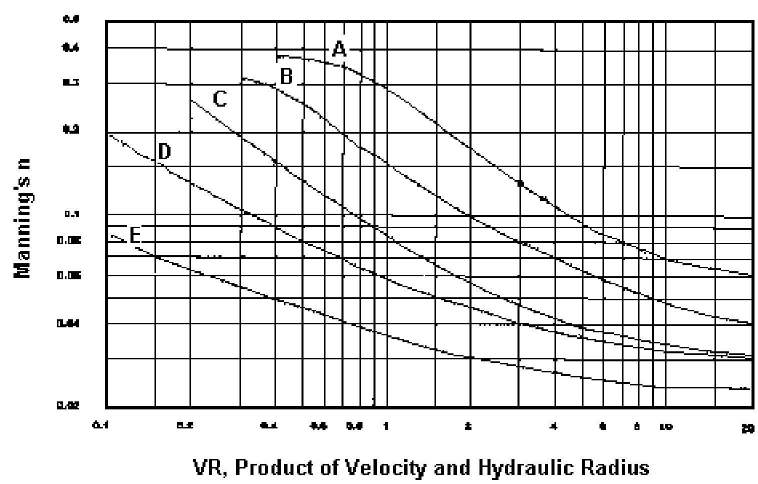

Figure 12.3 SCS Grass Cover n-value Curves (US Dept. of Agriculture, 1954)

Table 12-1
Characteristics of Grass Cover

Grass Type	Cover	Condition
A	Weeping lovegrass……………… Yellow bluestem Ischaemum……	Excellent Stand, tall (average 30 in) Excellent stand, tall (average 36 in)
B	Kudzu…………………………… Bermudagrass…………………… Native grass mixture (little bluestem, blue grama, other long and short Midwest grasses) Weeping lovegrass……………… Lespedeza serices………………. Alfalfa…………………………… Weeping lovegrass……………… Kudzu…………………………… Blue grama……………………….	Very dense growth, uncut Good stand, tall (average 12 in) Good stand, unmowed Good stand, tall (average 24 in) Good stand, not weedy, tall (average 19 in) Good stand, uncut (average 11 in) Good stand, mowed (average 13 in) Dense growth, uncut Good stand, uncut (average 13 in)
C	Crabgrass………………………... Bermudagrass…………………… Common lespedeza……………… Grass-legume mixture—summer (orchard grass, redtop, Italian ryegrass and common lespedeza) Centipedegrass…………………... Kentucky bluegrass………………	Fair stand, uncut (10 to 48 in) Good stand, mowed Good stand, uncut (average 11 in) Good stand, uncut (6 to 8 in) Very dense cover (average 6 in) Good stand headed (6 to 12 in)
D	Bermudagrass…………………… Common lespedeza……………… Buffalograss…………………….. Grass-legume mixture—fall, spring (orchard grass, redtop, Italian ryegrass and common lespedeza) Lespedeza serices……………….	Good stand, cut to 2.5 in height Excellent stand, uncut (average 4.5 in) Good stand, uncut (3 to 6 in) Good stand, uncut (4 to 5 in) After cutting to 2 in height; very good stand before cutting
E	Bermudagrass…………………… Bermudagrass……………………	Good stand, cut to 1.5 in height Burned stubble

Soil Conservation Service Grass Cover

The Soil Conservation Service (SCS, US Department of Agriculture, 1954) has developed five curves that define the respective roughness as a function of the product of velocity and hydraulic radius. Each curve, A through E, represents a different type of grass cover, all of which are presented in Table 12-1. The ranges over which these curves apply can be seen in Figure 12.3.

Selection of Roughness Equation

Each of the roughness equations described above have limitations to their applicability. Selection of one or more methods should be chosen based on stream characteristics with knowledge of the development of the chosen method(s) to better determine the appropriate roughness values to use. For example, vegetation roughness and bank angle typically do not permit the movement of bed load along the face of the banks, therefore bed roughness predictors such as Limerinos and Brownlie should not be used at those locations in the cross section. For this reason, HEC-RAS only allows the user to define one sediment gradation, which should be applied to the main channel bed only. In addition, the equations used to solve for Manning's n values are typically based on a representative grain diameter and hydraulic parameters. Other roughness affects such as vegetation, temperature, planform, etc., are not accounted for. The following table (Table 12.2) gives a general idea of the limitations and applicabilities of each roughness predictor.

Table 12-2
Data Range and Applicabilities of Roughness Predictors

Equation	Data Range	Applicability
Mannings	Typically $.01<n<.5$	All. However, n-values do not have the ability to directly vary with Hydraulic Radius
Keulegan	Froude number $0.2<F<8.0$	In streams where the relative roughness value, $R/k_s >= 3$
Strickler	$R/k_s >=1$	Natural channels for uniform flow computations.
Limerinos	$1.5mm<d_{84}<250mm$ $0.2<n<0.10$ $1ft<R<6ft$ $600<R/k_s<10,000$	Coarse sand to large cobbles. Only upper regime flow. Mobile beds. Main channel bed only.
Brownlie		Upper, lower, and transitional regimes. Mobile beds. Main channel bed only.
SCS Grass Curves	0.1 to $0.4<VR<20$	Grass cover. See Table 12-1

Stable Channel Design

Three approaches can be used in HEC-RAS for stable channel design. They are the Copeland, Regime, and Tractive Force methods. The Copeland method uses an analytical approach to solve stable channel design variables of depth, width, and slope. Stability is achieved when the sediment inflow to a particular reach equals the sediment outflow. The Regime method is purely empirical, and, within HEC-RAS, uses equations developed by Blench (1975). The Regime method defines a channel as being stable when there is no net annual scour or deposition in the design reach. The Tractive Force method is an analytical scheme that defines channel stability as no appreciable bed load movement. It is important to know the characteristics of the design stream to determine which approach will work best. Each of these approaches stem from work done previously in conditions with somewhat limited validity ranges.

Copeland Method

The Copeland Method for stable channel design was developed by Dr. Ronald Copeland at the Waterways Experiment Station for use in the SAM software package (Copeland, 1994). This approach is primarily analytical on a foundation of empirically-derived equations and it uses the sediment discharge and flow depth prediction methods of Brownlie (1981) to ultimately solve for stable depth and slope, for a given channel bottom width for trapezoidal cross sections. This method assumes bed load movement occurs above the bed, not the banks, and separates hydraulic roughness into bed and bank components.

To determine the level of stability of the design channel, an inflowing sediment discharge must be established. This can be done simply by entering the upstream sediment concentration, or by entering a supply reach bottom width and slope and allowing the program to calculate the sediment discharge. Sediment concentration is given by the following:

$$C = 9022(F_g - F_{go})^{1.978} S^{0.6601} \left(\frac{R_b}{d_{50}} \right)^{-0.3301} \qquad (12\text{-}21)$$

Where: C = Sediment concentration over the bed, in ppm
F_g = Grain-related Froude number
F_{go} = Critical grain-related Froude number
S = Slope
R_b = Bed hydraulic radius
d_{50} = Median grain size

$$F_g = \frac{V}{\sqrt{(s_s - 1)gd_{50}}} \qquad (12\text{-}22)$$

Where: V = Average channel velocity (this method assumes the average velocity for the total cross section is representative of the average velocity in each sub section).

s_s = Specific Gravity of sediment particles.

$$F_{go} = \frac{4.596\tau_{*o}^{0.5293}}{S^{0.1405}\sigma^{0.1606}} \qquad (12\text{-}23)$$

$$\tau_{*o} = 0.22Y + 0.06(10)^{-7.7Y} \qquad (12\text{-}24)$$

$$Y = \left(\sqrt{s_s - 1}R_g\right)^{-0.6} \qquad (12\text{-}25)$$

$$R_g = \sqrt{\frac{gd_{50}^3}{\nu}} \qquad (12\text{-}26)$$

$$\sigma = \frac{1}{2}\left(\frac{d_{84}}{d_{50}} + \frac{d_{50}}{d_{16}}\right) \qquad (12\text{-}27)$$

Where: τ_{*o} = Critical shear stress

Rg = Grain Reynolds Number

V = Kinematic viscosity

σ = Sediment gradation coefficient

Brownlie uses the above regression equations to equate critical shear from Shield's diagram with critical Froude number, which can ultimately be used to represent a critical velocity by substituting F_{go} into equation 12-22.
For the case where the Grain-related Froude Number is less than or equal to the Critical Grain-related Froude Number, the sediment concentration, C, will automatically be returned as zero, indicating no sediment bed movement.

Once the inflowing sediment concentration over the bed is determined, the total sediment concentration for the entire channel is used to size stable channel dimensions for various channel bottom widths. To do this, Brownlie's resistance equations are used:

$$R_b = 0.2836d_{50}q_*^{0.6248}S^{-0.2877}\sigma^{0.08013}\text{, for Upper Regime}$$

$$R_b = 0.3724d_{50}q_*^{0.6539}S^{-0.2542}\sigma^{0.1050}\text{, for Lower Regime} \qquad (12\text{-}28)$$

Where: q_* = dimensionless unit discharge
σ = sediment gradation coefficient

$$q_* = \frac{VD}{\sqrt{g d_{50}^3}} \tag{12-29}$$

Upper or lower transport regime is determined using the relationship expressed in equation 12-17. However, if the Grain-related Froude Number is within 0.8 to 1.25 of $1.74/S^{1/3}$, then it is considered to be in the transitional regime. Currently, a definition for a function describing the transitional transport regime is not available. The user has the choice of applying either the upper or lower regime equations in this circumstance. In the lower regime, the bed form can be composed of ribbons or ridges, ripples, dunes, bars, or simply a flat bed with transportation mostly as bed load. The transitional regime consists of washed-out dunes and sand waves, with particles transported mostly by suspension. The upper regime develops symmetrical sand waves in subcritical flow and plane bed and/or anti dunes for supercritical flow. Particles are almost entirely in suspension. If a transitional regime is realized in one or more of the solutions, recompute the stable channel dimensions using the other transport regime and compare results. Typically the upper regime is found on the rising end of a flood wave and the lower regime is found on the falling end. It is suggested that the more conservative results be used for design if the regime is not known.

Because the roughness of the side slopes is accounted for in this solution method, an assumption has to be made as to their hydraulic parameters. It is assumed that the average velocity over the side slopes is equal to the average channel velocity. With that,

$$R_s = \left(\frac{V n_s}{1.486 S^{0.5}} \right) \tag{12-30}$$

and the channel area, A, can be determined by

$$A = R_b P_b + R_s P_s \tag{12-31}$$

Where: R_s = Hydraulic radius of the side slopes
n_s = Manning's n value of the side slopes
P_s = Wetted perimeter of the side slopes
R_b = Hydraulic radius of the bed
P_b = Bed width.

The bed roughness is calculated using Brownlie's roughness predictor (Equation 12-19).

The user can enter a median channel width to bracket the desired results or this value can be left empty, in which case, HEC-RAS will automatically compute a median channel width from the following regime equation, which is proposed in EM 1110-2-1418:

$$B = 2.0Q^{0.5} \tag{12-32}$$

Using the median channel width, HEC-RAS determines 19 other channel widths at increments of 0.1B. Stable channel geometry is then solved for each channel width. A stability curve can be analyzed by plotting the array of base widths and their corresponding stable slopes within HEC-RAS by pressing the “Stability Curve” command button after computations have been run. As shown in Figure 12.4, it is easy to see for what slope/width channel geometries degradation, aggradation, or stabilization can be expected. It is important to note that the further away from the stability curve, the more aggradation of degradation can be expected. A second-order Lagrangian interpolation scheme is used to find the minimum stream power solution, which is the minimum slope that will transport the inflowing sediment load.

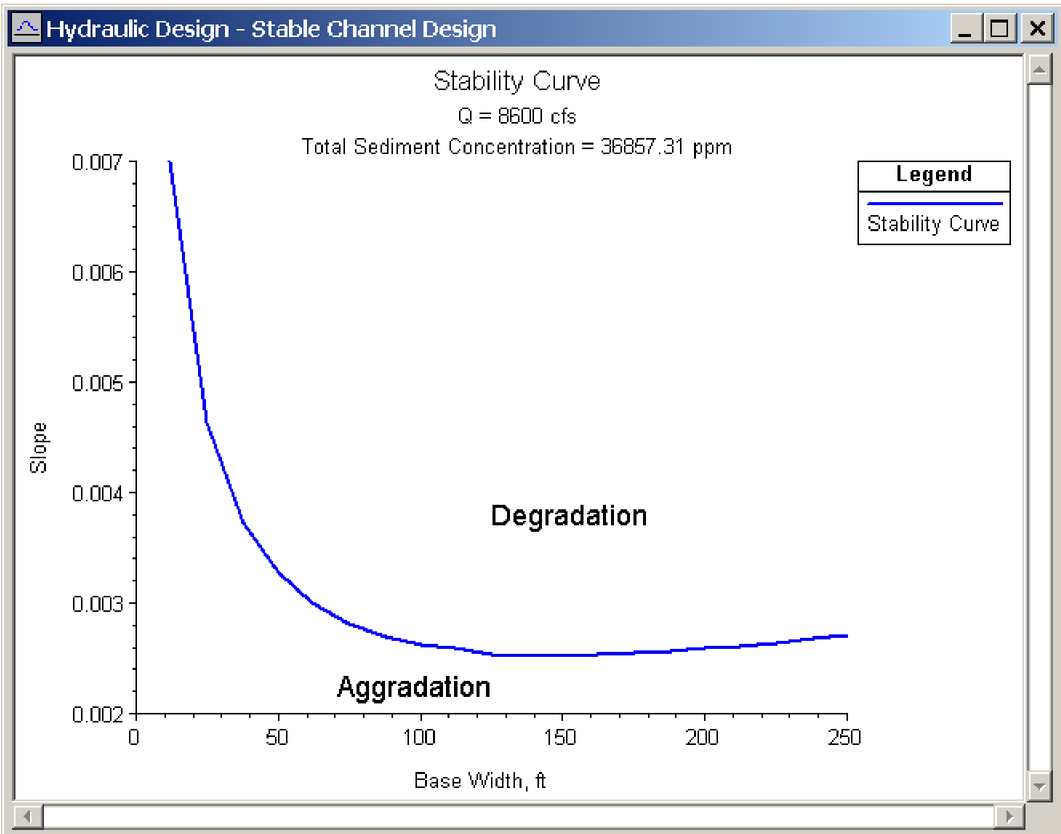

Figure 12.4 Stability Curve

The use of k values to define roughness on the side slopes is permitted for the Copeland Method. HEC-RAS simply converts the k value to an associated Manning’s n value using Strickler’s equation (Equation 12-15) with a value of 0.039 for the Strickler function, as suggested by Copeland. The bank roughness should be an accurate representation of everything that contributes to roughness on the banks. This includes channel irregularities, variations of channel cross-section shape, channel sinuosity, and vegetation. It is important to run the computations using a range of roughness values to test the sensitivity. Because, in this method, all sediment transport is assumed to occur over the bed, and not over the banks, flow distribution is very important for accuracy. This is accounted for in the bank steepness and roughness. For maximum transport, use a very steep bank with low roughness.

Sound judgment must be used when selecting the appropriate design discharge for performing a stability analysis. To date, no generally accepted discharge for stable channel design is agreed upon, therefore the use of a range of discharges is recommended. Suggested design discharges that may represent the channel forming discharge are:

- 2-year frequency flood (perennial streams)
- 10-year frequency flood (ephemeral streams)
- Bankfull discharge
- Effective discharge (Q that carries the most bed load sediment)

Selection of the design discharge should be made after considering the general physical characteristics of the stream, the temporal characteristics of the stream, what is the desired outcome (channel stabilization?), and any other applicable factor. It would be wise to run the calculations using a range of discharges as well as sediment inflows for a sensitivity analysis to understand how the channel reacts to different sediment and water inflow events.

As in the SAM package, HEC-RAS calculates a range of widths and slopes, and their unique solution for depth. This makes it possible to easily analyze or design stable channels. If a given slope is desired, the channel width through that reach can be adjusted to a value on the stability curve. Likewise, if a particular channel width is desired, the channel slope can be adjusted to achieve stability. If, for a given width, the slope is greater than the input valley slope, which is the maximum possible slope for the channel invert, this creates a sediment trap, which is indicate by the results. However, if the slope is less than the valley slope, the stability curve can be used to aid in adding sinuosity or the spacing of drop structures.

Because the Brownlie equations were developed from an analysis of field and laboratory data, there are limits of applicability that should be adhered to. At the least, the user needs to be aware if the limits are being exceeded. Table 12-3 presents the ranges of selected parameters of field and laboratory data used in Brownlie's research.

Table 12-3
Data Range and Applicabilities of Copeland Method

	Velocity (fps)		Depth (ft)		Slope x 10^3		d_{50} x 10^{-3} (ft)		Conc. (ppm)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Lab	0.73	6.61	0.11	1.91	0.269	16.950	0.28	4.42	10.95	39263
Field	1.20	7.95	0.35	56.7	0.010	1.799	0.28	4.72	11.70	5830

In addition, Brownlie suggests input data be restricted to the following:

Table 12-4
Suggested Input Restrictions for Copeland Method

Parameter	Symbol	Restiction	Reason
Median Grain Size (ft x 10^{-3})	d_{50}	$0.203<d_{50}<6.56$	Sand only
Geometric Standard Deviation of Bed Particles	σ_g	$\sigma_g < 5$	Eliminate bimodal grain distributions
Width to Depth Ratio	B/D	B/D > 4	Reduce sidewall effects
Relative Roughness	R_b/d_{50}	$R_b/d_{50} > 100$	Eliminate shallow water effects
Concentration (ppm)	C	C>10	Accuracy problems associated with low concentration

Regime Method

The regime method for stable channel design originated from irrigation design studies in Pakistan and India, and is based on a set of empirically derived equations, which typically solve for depth, width, and slope as a function of discharge and grain size.

$$D, B, S = f(Q, d_{50}) \tag{12-33}$$

Where: D = Depth
B = Channel width
S = Slope
Q = Discharge
D_{50} = median grain size.

To be considered in regime, or equilibrium, transport of sediments is allowed as long as there is no net annual scour or deposition in the channel. The regime method is applicable to large-scale irrigation systems with a wide range of discharges of silts and find sands. Because regime equations are purely empirical and based on field observations, the regime method can only be used within its validity range (Van Rijn, 1993).

The Blench Regime Method (Blench, 1970) is used in HEC-RAS. These equations are intended to be used with channels that have sand beds. In addition to the typical independent variables of discharge and grain size, the Blench method requires an inflowing sediment concentration and some information about the bank composition. The three regime equations are:

$$B = \left(\frac{F_B Q}{F_S}\right)^{0.5} \tag{12-34}$$

$$D = \left(\frac{F_S Q}{F_B^2}\right)^{\frac{1}{3}} \tag{12-35}$$

$$S = \frac{F_B^{0.875}}{\frac{3.63g}{\nu^{0.25}} B^{0.25} D^{0.125} \left(1 + \frac{C}{2330}\right)} \tag{12-36}$$

Where: D = Channel depth
B = Channel width
S = Channel slope
Q = Channel forming discharge
d_{50} = Median grain size of bed material
C = Bed material sediment concentration
ν = Kinematic viscosity
F_B = Bed factor
F_S = Side factor

The bed factor can be determined by the following equation:

$$F_B = 1.9\sqrt{d_{50}} \tag{12-37}$$

Blench suggests the following values be used for the side factor:

- F_S = 0.1, for friable banks
- F_S = 0.2, for silty, clayey, loamey banks
- F_S = 0.3, for tough clayey banks

The Blench regime method is applicable only to straight reaches with beds of silt to fine sand. In addition, Blench suggests that the regime equations be applied only under the following circumstances:

- Sides behave as if hydraulically smooth (i.e. friction due only to viscous forces).
- Bed width exceeds three times the depth.
- Side slopes are consistent with those of a cohesive nature.
- Discharges are steady.
- Sediment load is steady.
- Bed load is non-cohesive, and moves in dune formation.
- Subcritical flow.
- Sediment size is small compared with the depth of water.
- Regime has been achieved by the channel.

These circumstances seem very confining, and in reality, no one channel or canal can claim to behave strictly in this manner. However, if the channel can be adequately approximated by these conditions, without deviating significantly from its true nature, the regime equations may be applicable. At a minimum, the Blench Regime method is a quick way of obtaining "ball-park" figures for results.

Tractive Force Method

Essentially an analytical stable design method, the tractive force approach utilizes a critical shear stress to define when initiation of motion begins, the point at which the channel becomes unstable. In HEC-RAS, this concept is followed to allow the user to solve for two dependant variables when two others are given. The dependant variables are depth, width, slope, and a representative grain size (either d_{50} or d_{75}, depending on the solution method selected). For example, width and grain size can be entered, and HEC-RAS will solve for depth and slope.

The tractive force can be defined as the force that is resisted by friction force and, while in equilibrium, is equal and opposite in magnitude and direction. It is also called shear stress or drag force and can be represented as:

$$\tau_o = \gamma R S \tag{12-38}$$

Where: τ_o = Tractive force per unit wetted area
γ = Unit weight of water
R = Hydraulic radius
S = Slope

For very wide channels (B/D > 10), equation 12-38 is very representative of the shearing force felt on the bed. Because τ_o is the average tractive force over the wetted area, the shear distribution becomes more non-uniform as the channel becomes narrower and more trapezoidal. As a result, the maximum tractive force is actually less than that predicted by equation 12-38 by some reduction factor. In addition, the channel walls, due to their inclination, have an even greater reduction effect on the maximum tractive force felt on the side slopes. For typical trapezoidal sections, it has been determined experimentally by Lane (1953) that the adjustment factor for both the bed and side slopes is largely dependent on the width to depth ratio and the side slope angle. Figure 12.5 presents the curves used to determine the adjustment factors for both the bed and side slopes.

The channel is considered stable if the tractive force at any given location in the cross section is less than the critical shear force. There are currently three methods for determining the critical shear stress in HEC-RAS. They are the Lane, Shields, and user-entered methods.

Lane Method:

Lane conducted experiments on canals in the San Luis Valley of Colorado to develop a method for predicting the critical shear stress. The canals tested were stable, straight, and regular in section, with a wide range of coarse particle sizes from about 0.3 inches to 3 inches in diameter. The results

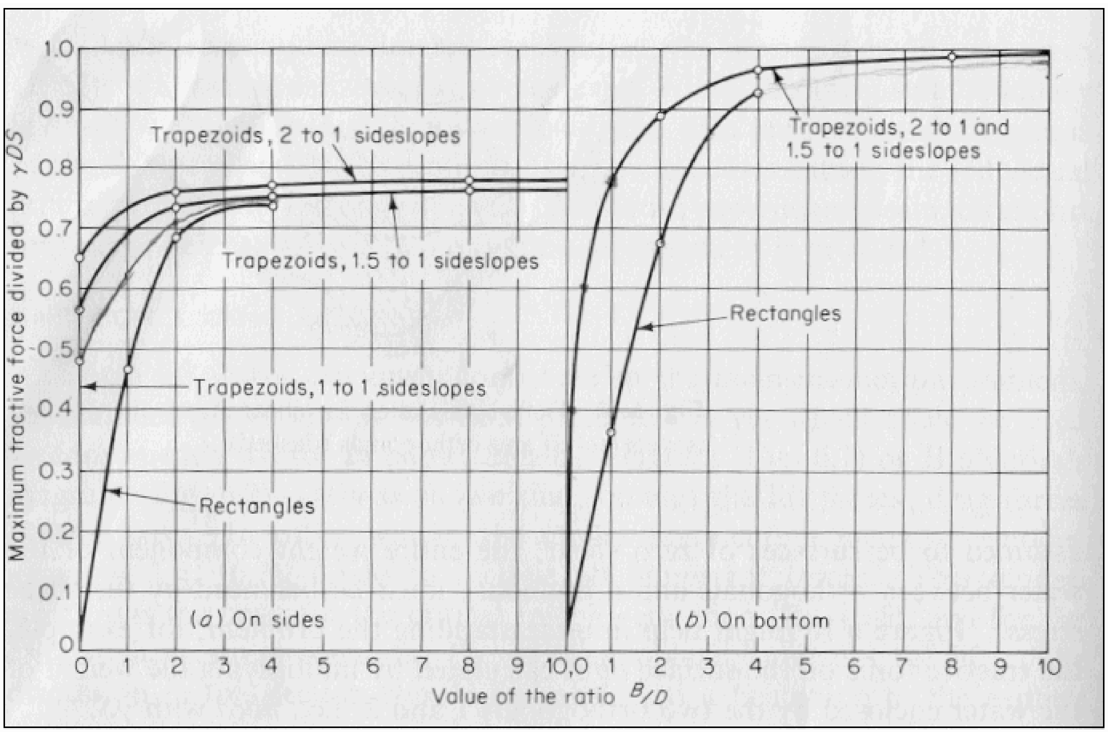

Figure 12.5 Maximum Shear Stress in a Channel (Lane, 1953)

indicated that the critical shear stress was more or less linearly related to the diameter of the particle as follows:

$$\tau_{cr} = 0.4 d_{75} \tag{12-39}$$

The particle size, d_{75}(inches) was used because Lane noticed that throughout the experiments, the smaller particles were consistently shielded by the larger ones. By using a particle size in which only 25% of the particles were larger by weight, the initiation of motion was better represented.

The Shields method has historically been much more widely used to determine the initiation of motion. Shields (1936) developed a relationship between the shear Reynolds number, Re_* and the critical mobility parameter, θ_{cr} from a wide range of experimental data. Shield's diagram is presented in Figure 12.6. The Shear Reynolds number is a representation of the ratio of inertial forces to viscous forces *at the bed* and is given as:

$$\mathrm{Re}_* = \frac{u_* d}{\nu} \tag{12-40}$$

Where: u_* = Shear velocity, which is a representation of the intensity of turbulent fluctuations in the boundary layer.
d = Representative particle size (d_{50} is used in HEC-RAS)
ν = Kinematic viscosity

$$u_* = \sqrt{gDS} \tag{12-41}$$

Where: D = Water depth
S = Channel slope

The critical mobility parameter is also known as the dimensionless shear stress and is given as:

$$\theta_{cr} = \frac{\tau_{cr}}{(\gamma_s - \gamma)d} \tag{12-42}$$

Where: γ_s = Unit weight of the particles
γ = Unit weight of water

From reviewing Shield's diagram, a number of things become clear. First, it is evident that the critical mobility parameter never drops below about 0.03. If the specific gravity of the sediments and the unit weight of water are assumed to be 2.65 and 62.4 lb/ft^3, respectively, then the critical shear stress in lb/ft^2 is never less than about 3 times the particle diameter (in feet). Also, if the shear Reynolds number exceeds about 450, the viscous forces in the sublayer no

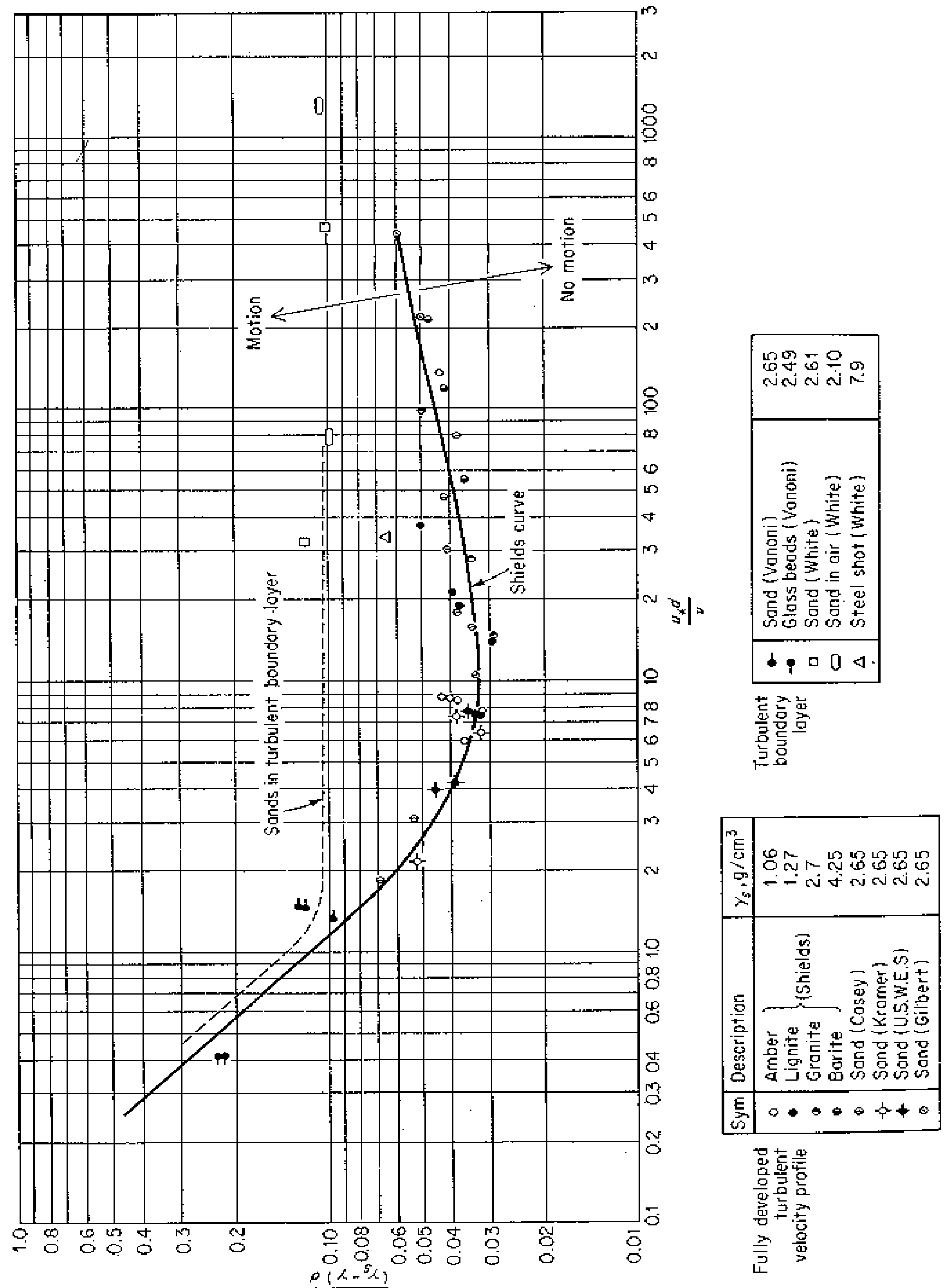

Figure 12.6 Shield's Diagram, Graf (1971)

longer have an effect on the shearing force and the Shield's curve levels off with a critical mobility parameter of about 0.055. At this point, the critical shear stress is purely a function of the particle characteristics (size, weight). Likewise, when the shear Reynolds number drops below about 2.0, the inertial forces in the sublayer are negligible and the critical shear stress becomes linearly related to the particle characteristics and the inverse of the viscosity. However, in most natural stream conditions, the shear Reynolds number is high and inertial forces are dominant. HEC-RAS, however, will solve for the critical mobility parameter throughout the full range of Shield's diagram.

A third solution option provided in HEC-RAS allows the user to enter in a value for the critical mobility parameter. This option is given due to the wide range of research on initiation of motion and the varying definitions of what exactly *initiation of motion* means. Although the Shield's curve is meant to represent the initiation of motion, more recent research indicates that this curve more accurately represents permanent grain movement at all locations of the bed. This can be quite different from the shearing required to initiate motion of one or a few particles. Figure 12.7 presents the Shield's curve overlain on seven qualitative curves developed by Delft Hydraulics (1972) describing particle movement. It is evident that the critical shear stress found with Shield's curve can be as much as twice the value required to cause occasional particle movement at some locations.

Because of the variety of opinions on this matter, the user is able to supply HEC-RAS with his/her own value for the critical mobility parameter. This value should be selected such that it represents not only the type of conditions present, but also the type of results desired (i.e. is the design based on permanent particle movement, infrequent particle movement, no particle movement, total suspension, etc?). Many curves present the critical shear stress as the dependent parameter in the initiation of motion curves. A collection of these types of curves is shown in Figure 12.8. **It is important for the user to know that the value entered into RAS must be in the form of the Critical Mobility Parameter, or dimensionless shear stress shown as equation 12-42**.

In HEC-RAS, a reduction factor is applied to the critical shear stress on the side slopes to account for the greater effect of gravity on the particle stability.

$$\tau_{cr,s} = k_{\alpha}\tau_{cr} \tag{12-43}$$

Where: $\tau_{cr,s}$ = Critical shear stress on the side slope
τ_{cr} = Critical shear stress on the bed
k_{α} = Reduction factor

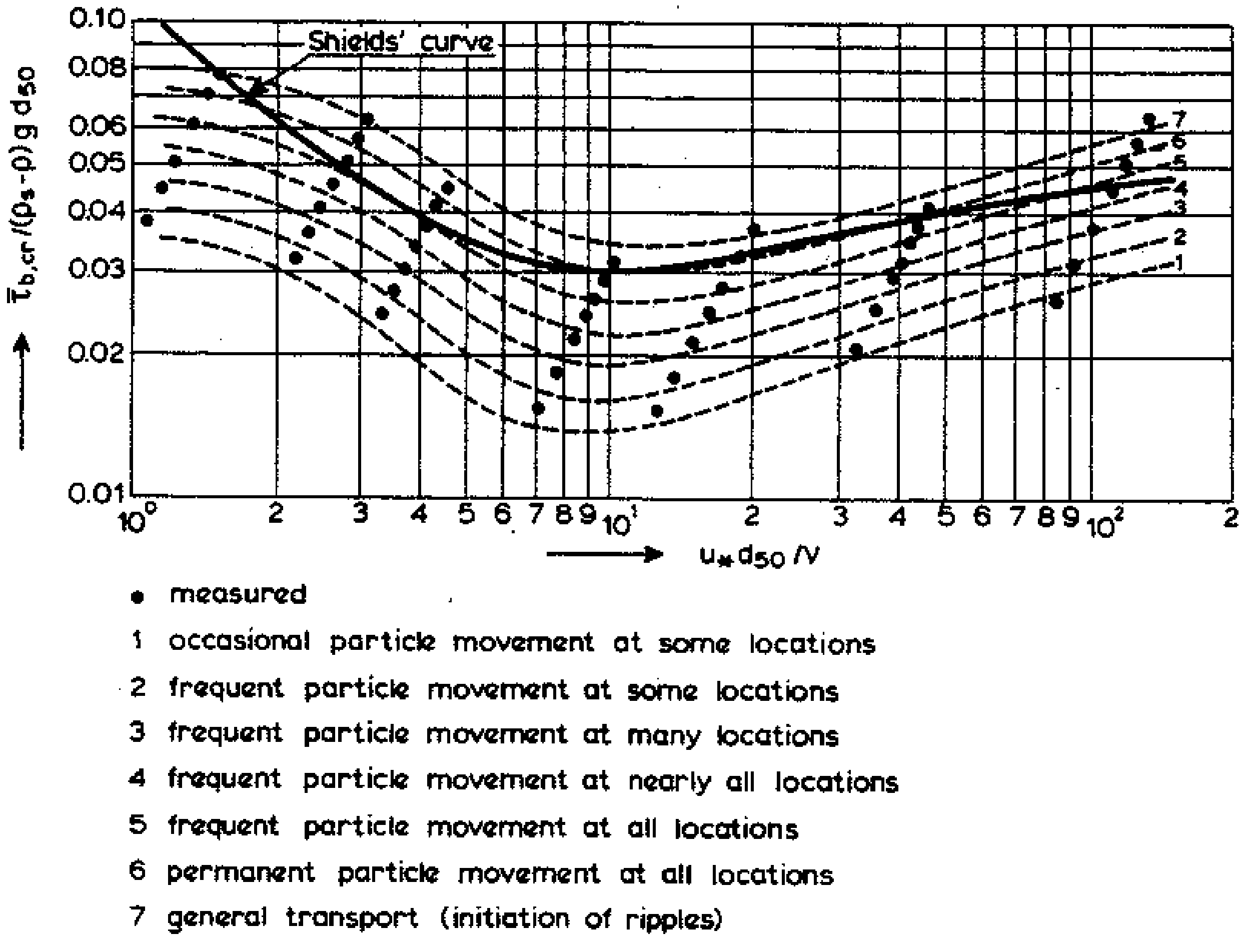

Figure 12.7 Initiation of Motion and Suspension for a Current Over a Plane Bed (Delft Hydraulics, 1972)

$$k_\alpha = \cos\alpha\sqrt{1-\frac{\tan^2\alpha}{\tan^2\phi}} \qquad (12\text{-}44)$$

Where: α = Angle of the side slope, in degrees

ϕ = Angle of repose of the sediment, in degrees

and $\phi > \alpha$

The angle of repose of the sediment particles must be entered by the user for the bed and both of the side slopes. Lane provides a diagram that suggests values for angles of repose for different grain sizes and angularities (Figure 12.9).

HEC-RAS allows the user to solve for two dependant variables when two others are provided. The computations equate the critical shear stress with the actual shear stress to solve the first variable and then uses Manning's equation to solve the second variable. If the particle size is to be computed by HEC-RAS, one or all of the particle sizes (bed, left side slope, or right side slope) can be solved for, along with one other variable (depth, slope, or width). The equation RAS uses to determine the two unknown variables depends on the two unknown variables selected. Particle size is always determined using tractive force (i.e. equating critical shear with actual shear). The following table (Table 12-5) indicates which variable is solved by which method. This is helpful to know, in order to make sense of the results.

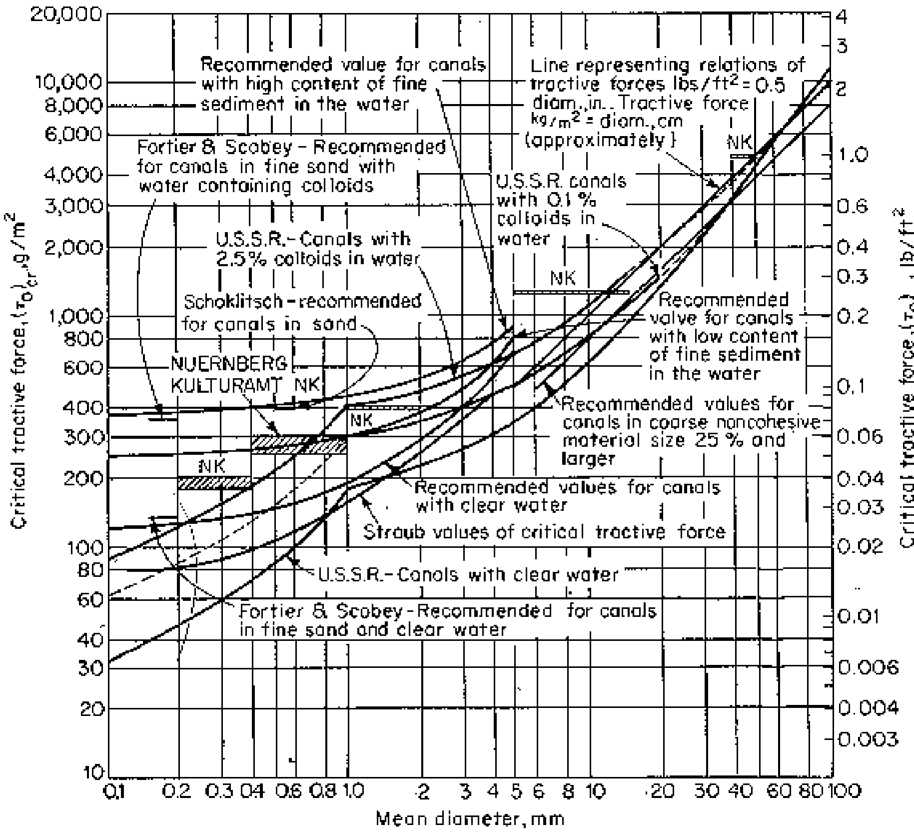

Figure 12.8 Critical shear stress as a function of grain diameter (Lane, 1953)

For example, assume depth and width are to be solved for. If a large diameter grain size is used, a high value for allowable depth will be returned by the tractive force equations. Then because this depth is high, Manning's equation will return a very low value for width, sometimes unrealistic. Be aware that *the value for width is the value to achieve uniform flow based on the maximum allowable depth for a stable cross section*. The variables "width" and "maximum depth" in the above statement can be replaced with any of the four dependant variables in accordance with the equation priorities as shown in Table 12-5.

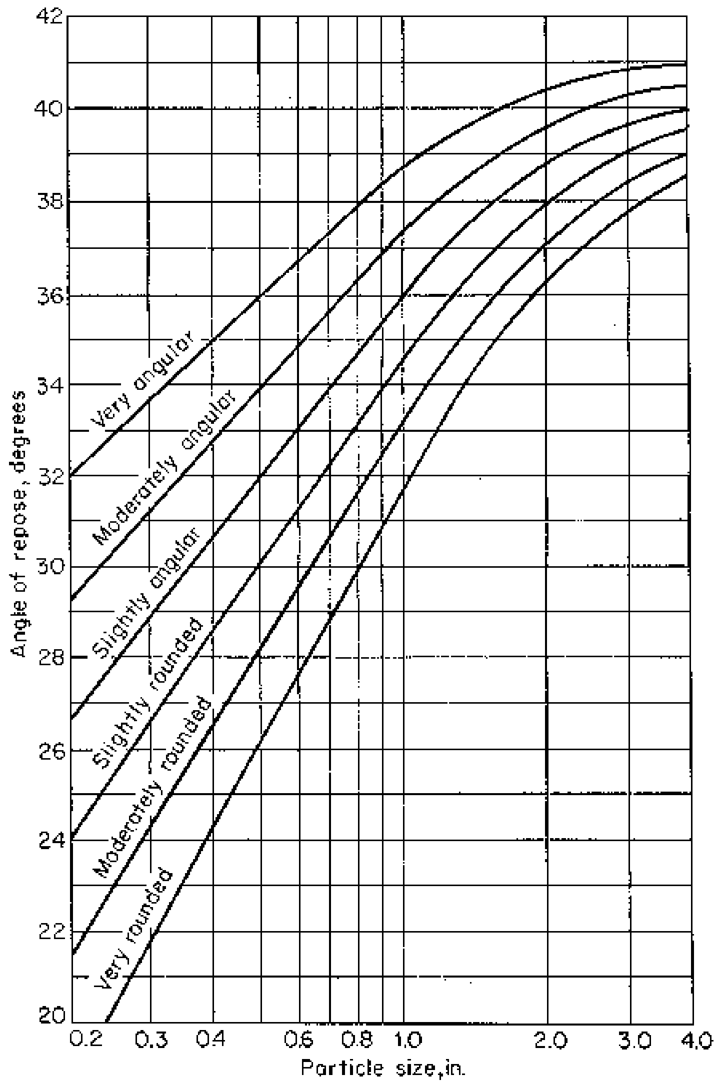

Figure 12.9 Angle of Repose for Non-Cohesive Material (Lane, 1953)

The result of this solution technique can create an apparent inconsistency that the user must be aware of. If width and slope are solved for, slope will be determined by tractive force and width will be determined by Mannings. Now if the resulting width is used to solve for slope and particle size, the particle size will be different from what was used in the first solution. This is because when particle size and slope are solved for, particle size is first solved for using tractive force, then slope is solved using *Mannings*. Because true uniform flow conditions are rarely found on river reaches, be sure that the tractive force method is the equation solving the variable you are most interested in.

For more information on all three stable channel design methods presented herein, refer to the referenced literature.

Table 12-5
Solution Priorities for Tractive Force Method

Unknown Variables	Tractive Force	Mannings
d, D	Min d	D
d, B	Min d	B
d, S	Min d	S
D, B	Max D	B
D, S	Max D	S
B, S	Max S	B

Where: d = particle size (d_{50} for Shields, d_{75} for Lane)
D = Depth
B = Width
S = Slope

Sediment Transport Capacity

The sediment transport capacity function in HEC-RAS has the capability of predicting transport capacity for non-cohesive sediment at one or more cross sections based on existing hydraulic parameters and known bed sediment properties. It does not take into account sediment inflow, erosion, or deposition in the computations. Classically, the sediment transport capacity is comprised of both bed load and suspended load, both of which can be accounted for in the various sediment transport predictors available in HEC-RAS. Results can be used to develop sediment discharge rating curves, which help to understand and predict the fluvial processes found in natural rivers and streams.

Background

Transported sediment is comprised of bed load, suspended load, and wash load. Van Rijn (1993) defines them as:

<u>Suspended load</u>: That part of the total sediment transport which is maintained in suspension by turbulence in the flowing water for considerable periods of time without contact with the streambed. It moves with practically the same velocity as that of the flowing water.

<u>Bed load</u>: The sediment in almost continuous contact with the bed, carried forward by rolling, sliding, or hopping.

<u>Wash load</u>: That part of the suspended load which is composed of particle sizes smaller than those found in appreciable quantities in the bed material. It is in near-permanent suspension and, therefore, is transported through the stream without deposition. The discharge of the wash load through a reach depends only on the rate with which these particles become available in the catchment area and not on the transport capacity of the flow.

Because wash load volume is purely a function of the upstream catchment and not the study reach, it is ignored in the sediment transport computations. However, a particle size considered wash load at one cross section in a reach, may become suspended load at a downstream section, and eventually may become bed load. Therefore, it is important to account for the wash load in a system-wide sediment analysis.

The initiation of motion of particles in the bed depends on the hydraulic characteristics in the near-bed region. Therefore, flow characteristics in that region are of primary importance. Since determining the actual velocity at the bed level is difficult, particularly with 1-D model results, shear stress has become the more prevalent, though not exclusive, way of determining the point of incipient motion. Shear stress at the bed is represented by the following:

$$\tau_b = \gamma RS \qquad \text{12-45}$$

Where: τ_b = Bed shear stress
γ = Unit weight of water
R = Hydraulic radius
S = Energy slope

Another factor that plays an important role in the initiation and continued suspension of particles is the turbulent fluctuations at the bed level. A measure of the turbulent fluctuations near the bed can be represented by the current-related bed shear velocity:

$$u_* = \sqrt{\frac{\tau_b}{\rho}} \quad \text{or} \quad u_* = \sqrt{gRS} \qquad \text{12-46}$$

Where: u_* = Current-related bed shear velocity

Additionally, the size, shape, roughness characteristics, and fall velocity of the representative particles in the stream have a significant influence on their ability to be set into motion, to remain suspended, and to be transported. The particle size is frequently represented by the median particle diameter (d_m). For convenience, the shape is typically represented as a perfect sphere, but sometimes can be accounted for by a shape factor, and the roughness is a function of the particle size.

In general, a typical sediment transport equation for multiple grain size classes can be represented as follows:

$$g_{si} = f\left(D, V, S, B, d, \rho, \rho_s, sf, d_i, p_i, T\right) \qquad \text{12-47}$$

Where: g_{si} = Sediment transport rate of size class i
D = Depth of flow
V = Average channel velocity
S = Energy slope
B = Effective channel width
d = Representative particle diameter
ρ = Density of water
ρ_s = Density of sediment particles
sf = Particle shape factor
d_i = Geometric mean diameter of particles in size class i
p_i = Fraction of particle size class i in the bed.
T = Temperature of water

Not all of the transport equations will use all of the above parameters. Typically one or more correction factors (not listed) are used to adapt the basic formulae to transport measurements. Refer to the respective references for more detail.

Fall Velocity

The suspension of a sediment particle is initiated once the bed-level shear velocity approaches the same magnitude as the fall velocity of that particle. The particle will remain in suspension as long as the vertical components of the bed-level turbulence exceed that of the fall velocity. Therefore, the determination of suspended sediment transport relies heavily on the particle fall velocity.

Within HEC-RAS, the method for computing fall velocity can be selected by the user. Three methods are available and they include Toffaleti (1968), Van Rijn (1993), and Rubey (1933). Additionally, the default can be chosen in which case the fall velocity used in the development of the respective sediment transport function will be used in RAS. Typically, the default fall velocity method should be used, to remain consistent with the development of the sediment transport function, however, if the user has specific information regarding the validity of one method over the other for a particular combination of sediment and hydraulic properties, computing with that method is valid. The shape factor (sf) is more important for medium sands and larger. Toffaleti used a sf of 0.9, while Van Rijn developed his equations for a sf of 0.7. Natural sand typically has a sf of about 0.7. The user is encouraged to research the specific fall velocity method prior to selection.

$$sf = \frac{c}{\sqrt{ab}} \qquad \text{12-48}$$

Where: a = Length of particle along the longest axis perpendicular to the other two axes.

b = Length of particle along the intermediate axis perpendicular to other two axes.

c = Length of particle along the short axis perpendicular to other two axes.

Toffaleti: (Toffaleti, 1968). Toffaleti presents a table of fall velocities with a shape factor of 0.9 and specific gravity of 2.65. Different fall velocities are given for a range of temperatures and grain sizes, broken up into American Geophysical Union standard grain size classes from Very Fine Sand (VFS) to Medium Gravel (MG). Toffaleti's fall velocities are presented in Table 12.6.

Van Rijn: (Van Rijn, 1993). Van Rijn approximated the US Inter-agency Committee on Water Resources' (IACWR) curves for fall velocity using non-spherical particles with a shape factor of 0.7 in water with a temperature of 20^{o}C. Three equations are used, depending on the particle size:

$$\omega = \frac{(s-1)gd}{18\nu} \quad 0.001 < d \le 0.1 \text{ mm} \qquad 12\text{-}49$$

$$\omega = \frac{10\nu}{d}\left[\left(1 + \frac{0.01(s-1)gd^3}{\nu^2}\right)^{0.5} - 1\right] \qquad 0.1 < d < 1 \text{ mm} \qquad 12\text{-}50$$

$$\omega = 1.1\left[(s-1)gd\right]^{0.5} \qquad d \ge 1 \text{ mm} \qquad 12\text{-}51$$

Where: ω = Particle fall velocity
ν = Kinematic viscosity
s = Specific gravity of particles
d = Particle diameter

Table 12.6
Fall Velocity (Toffaleti, 1968)

Sand Grain Settling Velocity Versus Temperature, SP.G. 2.65, Shape Factor 0.9

TEMP °F	SETTLING VELOCITY IN FT/SEC								TEMP °F	SETTLING VELOCITY IN FT/SEC							
	VFS	FS	MS	CS	VCS	VFG	FG	MG		VFS	FS	MS	CS	VCS	VFG	FG	MG
35	.013	.045	.130	.305	.590	1.00	1.41	1.95	65	.021	.065	.165	.354	.640	1.00	1.41	1.95
36	.013	.045	.131	.307	.592	1.00	1.41	1.95	66	.021	.066	.166	.356	.641	1.00	1.41	1.95
37	.013	.046	.132	.310	.594	1.00	1.41	1.95	67	.021	.067	.167	.357	.643	1.00	1.41	1.95
38	.014	.047	.133	.312	.596	1.00	1.41	1.95	68	.022	.067	.168	.359	.644	1.00	1.41	1.95
39	.014	.047	.135	.314	.598	1.00	1.41	1.95	69	.022	.068	.170	.360	.646	1.00	1.41	1.95
40	.014	.048	.136	.316	.600	1.00	1.41	1.95	70	.022	.069	.171	.361	.647	1.00	1.41	1.95
41	.015	.049	.137	.318	.602	1.00	1.41	1.95	71	.022	.070	.172	.362	.649	1.00	1.41	1.95
42	.015	.050	.138	.320	.604	1.00	1.41	1.95	72	.023	.071	.173	.363	.650	1.00	1.41	1.95
43	.015	.051	.140	.321	.606	1.00	1.41	1.95	73	.023	.071	.175	.364	.652	1.00	1.41	1.95
44	.016	.051	.141	.322	.608	1.00	1.41	1.95	74	.023	.072	.176	.365	.653	1.00	1.41	1.95
45	.016	.052	.142	.323	.609	1.00	1.41	1.95	75	.024	.072	.177	.366	.655	1.00	1.41	1.95
46	.016	.053	.143	.325	.610	1.00	1.41	1.95	76	.024	.073	.178	.367	.656	1.00	1.41	1.95
47	.016	.053	.144	.326	.612	1.00	1.41	1.95	77	.024	.073	.180	.368	.657	1.00	1.41	1.95
48	.017	.054	.145	.328	.614	1.00	1.41	1.95	78	.024	.074	.181	.370	.658	1.00	1.41	1.95
49	.017	.055	.146	.330	.616	1.00	1.41	1.95	79	.025	.074	.182	.371	.659	1.00	1.41	1.95
50	.017	.055	.147	.331	.618	1.00	1.41	1.95	80	.025	.075	.183	.373	.660	1.00	1.41	1.95
51	.018	.056	.148	.333	.620	1.00	1.41	1.95	81	.025	.075	.184	.375	.661	1.00	1.41	1.95
52	.018	.057	.150	.334	.621	1.00	1.41	1.95	82	.025	.076	.185	.376	.662	1.00	1.41	1.95
53	.018	.057	.151	.336	.623	1.00	1.41	1.95	83	.025	.077	.186	.378	.663	1.00	1.41	1.95
54	.018	.058	.152	.338	.624	1.00	1.41	1.95	84	.026	.077	.187	.380	.664	1.00	1.41	1.95
55	.018	.059	.153	.340	.626	1.00	1.41	1.95	85	.026	.078	.188	.381	.665	1.00	1.41	1.95
56	.019	.059	.154	.341	.627	1.00	1.41	1.95	86	.026	.078	.190	.383	.666	1.00	1.41	1.95
57	.019	.060	.155	.343	.629	1.00	1.41	1.95	87	.026	.079	.192	.385	.667	1.00	1.41	1.95
58	.019	.061	.156	.344	.630	1.00	1.41	1.95	88	.027	.079	.194	.386	.668	1.00	1.41	1.95
59	.019	.061	.157	.346	.632	1.00	1.41	1.95	89	.027	.080	.195	.388	.669	1.00	1.41	1.95
60	.020	.062	.158	.347	.633	1.00	1.41	1.95	90	.027	.080	.196	.390	.670	1.00	1.41	1.95
61	.020	.063	.160	.349	.635	1.00	1.41	1.95	91	.028	.081	.197	.391	.671	1.00	1.41	1.95
62	.020	.063	.161	.350	.636	1.00	1.41	1.95	92	.028	.081	.198	.392	.672	1.00	1.41	1.95
63	.020	.064	.162	.351	.638	1.00	1.41	1.95	93	.028	.082	.199	.393	.673	1.00	1.41	1.95
64	.021	.065	.163	.353	.639	1.00	1.41	1.95	94	.028	.082	.200	.394	.674	1.00	1.41	1.95

Rubey: (Rubey, 1933). Rubey developed an analytical relationship between the fluid, sediment properties, and the fall velocity based on the combination of Stoke's law (for fine particles subject only to viscous resistance) and an impact formula (for large particles outside the Stoke's region). This equation has been shown to be adequate for silt, sand, and gravel grains. Rubey suggested that particles of the shape of crushed quartz grains, with a specific gravity of around 2.65, are best applicable to the equation. Some of the more cubic, or uniformly shaped particles tested, tended to fall faster than the equation predicted. Tests were conducted in water with a temperature of 16^o Celsius.

$$\omega = F_1\sqrt{(s-1)gd_s} \qquad 12\text{-}52$$

in which $F_1 = \sqrt{\frac{2}{3}+\frac{36\nu^2}{gd^3(s-1)}} - \sqrt{\frac{36\nu^2}{gd^3(s-1)}}$ 12-53

Correction for Fine Sediment

The viscosity of a fluid has a significant affect on the fall velocity of a particle within that fluid. In clear water, the kinematic viscosity is on the order of 1 X 10^{-5} ft^2/s, however, when a high concentration of fine sediment, particularly clay particles, is present, the viscosity will increase, in much the same way as when the water temperature is reduced. Colby (1964) proposed an adjustment factor to account for high concentration of fines, as well as temperature, which is shown in Figure 12.10.

HEC-RAS provides and field for the user to enter the concentration of fine sediments. This is an *optional* field, and, if left blank, bypasses the Colby adjustment factor calculations. Concentration magnitudes are entered in parts per million (ppm).

Sediment Gradation

Sediment transport rates are computed for the prescribed hydraulic and sediment parameters for each representative grain size. Transport capacity is determined for each grain size as if that particular grain size made up 100% of the bed material. The transport capacity for that size group is then multiplied by the fraction of the total sediment that that size represents. The fractional transport capacities for all sizes are summed for the total sediment transport capacity.

$$g_s = \sum_{i=1}^{n} g_{si}p_i \qquad 12\text{-}54$$

Where: g_s = Total sediment transport
g_{si} = Sediment transport for size class i
p_i = Fraction of size class i in the sediment
n = Number of size classes represented in the gradation

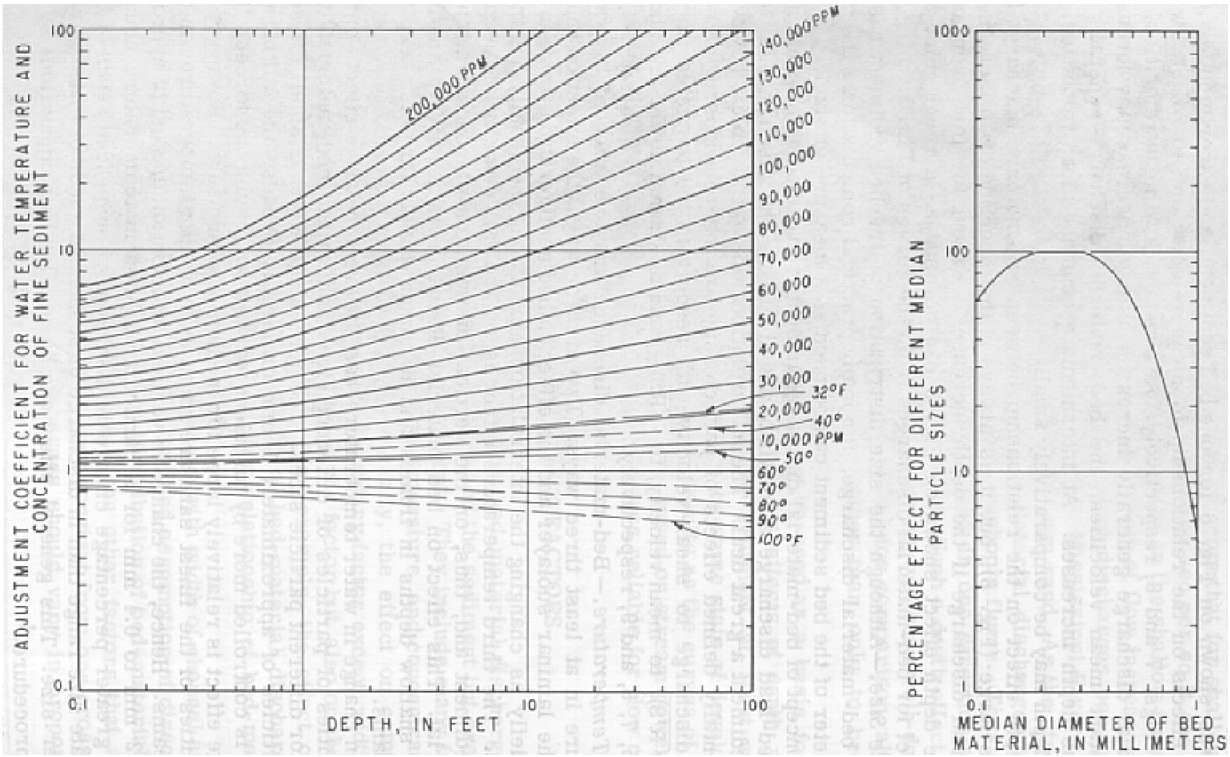

Figure 12.10 Adjustment Factor for Concentration of Fine Sediment (Colby, 1964)

The user enters gradation information as particle sizes with an associated percentage value that indicates the amount of material within the sediment mixture that is finer by volume (percent finer). HEC-RAS then interpolates logarithmically to determine a representative percent finer for the standard grade class sizes. The standard grade class sizes are based on the American Geophysical Union (AGU) classification scale shown in Table 12-6.

If a maximum particle diameter is not entered (i.e. d_{100}), HEC-RAS will automatically assign the 100% finer value to the next greater standard grain size from the largest particle diameter established by the user. For example, if the largest particle diameter is entered as 1.6 mm with a percent finer value of 84%, then the maximum grain size will be automatically assigned to 2.0 mm with 100% of the particles finer than that. On the low end, if the user does not establish a zero percent finer particle diameter (i.e. d_0), then the smallest standard grain size range (0.002 – 0.004 mm) is assigned zero percent. Because the ultra-fine sized sediment has a tendency to produce inaccurate results for certain transport functions, it is important that the user realize the extrapolation used in this instance. To avoid the automatic extrapolation on the fine-side of the gradation curve, simply enter in a particle diameter with an associated "percent finer" value of zero.

Table 12-6
Grain Size Classification of Sediment Material
American Geophysical Union

Sediment Material	Grain Diameter Range(mm)	Geometric Median Diameter (mm)
Clay	0.002-0.004	0.003
Very Fine Silt	0.004-0.008	0.006
Fine Silt	0.008-0.016	0.011
Medium Silt	0.016-0.032	0.023
Coarse Silt	0.032-0.0625	0.045
Very Fine Sand	0.0625-0.125	0.088
Fine Sand	0.125-0.250	0.177
Medium Sand	0.250-0.5	0.354
Coarse Sand	0.5-1.0	0.707
Very Coarse Sand	1-2	1.41
Very Fine Gravel	2-4	2.83
Fine Gravel	4-8	5.66
Medium Gravel	8-16	11.3
Coarse Gravel	16-32	22.6
Very Coarse Gravel	32-64	45.3
Small Cobbles	64-128	90.5
Large Cobbles	128-256	181
Small Boulders	256-512	362
Medium Boulders	512-1024	724
Large Boulders	1024-2048	1448

If the user enters in one or more particle sizes that are less than the smallest standard grain size diameter (0.002 mm), HEC-RAS will automatically lump all of that sediment into the smallest standard grain size range (Clay, 0.002 to 0.004 mm). This is done so that all of the sediment in the gradation curve will be accounted for volumetrically.

The rate of transport is extremely sensitive to the grain size distribution, particularly on the finer side, and should be chosen carefully. The application of grain size particles smaller than the designated range of applicability for a given function can lead to extremely high, and unreasonable sediment transport rates. For this reason, RAS provides an option to not compute sediment transport rates for grain sizes outside the range of applicability on the low end. This is done by going to the options menu and selecting "No" under the menu item "Compute for Small Grains Outside Applicable Range". Still, the user should check unreasonable results for **all** given parameter ranges (Table 12.7). (Note: the low end of applicable grain size for Laursen was chosen as that used in the *field* research.) The selection of a representative sediment sampling is described in EM 1110-2-4000.

Hydraulic Parameters

The hydraulic parameters used to compute sediment transport capacity are taken from the output of steady or unsteady flow runs. The user is required only to indicate for which profile the sediment transport computations will be made for each sediment reach. HEC-RAS automatically retrieves the required hydraulic input parameters, depending on which sediment transport function has been selected. **Therefore, steady, or unsteady flow computations must be run before sediment capacity computations can be performed.** The hydraulic parameters are retrieved from the steady output computations for the left overbank, main channel, and right overbank, as defined by the *sediment bank stations*. The total sediment transport for the cross section is then the sum of the three sub-sections.

Because different sediment transport functions were developed differently with a wide range of independent variables, HEC-RAS gives the user the option to select how depth and width are to be computed. The HEC-6 method converts everything to an effective depth and width by the following equations:

$$EFD = \frac{\sum_{i=1}^{n} D_{avg} a_i D_{avg}^{\frac{2}{3}}}{\sum_{i=1}^{n} a_i D_{avg}^{\frac{2}{3}}} \qquad EFW = \frac{\sum_{i=1}^{n} a_i D_{avg}^{\frac{2}{3}}}{EFD^{\frac{5}{3}}} \qquad \text{12-55/6}$$

Where: EFD = Effective depth
EFW = Effective width
a_i = Area of subsection i
D_{avg} = Average depth of sub section i
n = Number of subsections

However, many of the sediment transport functions were developed using hydraulic radius and top width, or an average depth and top width. For this reason, HEC-RAS allows the user to designate which depth/width method to use. If the default selection is chosen, then the method consistent with the development of the chosen function will be used. For irregular cross section shapes, RAS uses the effective depth/effective width or hydraulic radius/top width as the default. Also available for use is the hydraulic depth, which is used to represent the average depth and is simply the total area of the section divided by the top width. RAS computes these depth/width parameters for the left overbank, main channel, and right overbank, as designated by the bed load stations.

Bed Load Stations

By default, the channel bank stations are used to separate the left overbank, main channel, and right overbank for sediment transport computations. However, this may not necessarily represent the sediment distribution across the cross section. Therefore, HEC-RAS allows the user to designate bed load stations to separate the three channels based on sediment properties.

Output

HEC-RAS provides the option of viewing results in sediment rating curves and profile plots. The rating curve plot presents the sediment transport capacity vs. the river discharge and can be plotted for one or more cross sections. The profile plot presents the sediment transport capacity along the stream length for one or more sediment reaches.

Both types of plots allow have a number of dropdown boxes that allow the user to specify what is required for plotting. For example, by default, the total sediment transport rate is given for each cross section when a plot is opened. However, the user can view just the sediment transport of a single grain size or can compare sediment transport capacities of two or more grain sizes. Additionally, the user has the ability to view the overbanks and main channel separately as well as each transport function.

Sediment Transport Functions

Because different sediment transport functions were developed under different conditions, a wide range of results can be expected from one function to the other. Therefore it is important to verify the accuracy of sediment prediction to an appreciable amount of measured data from either the study stream or a stream with similar characteristics. It is very important to understand the processes used in the development of the functions in order to be confident of its applicability to a given stream.

Typically, sediment transport functions predict rates of sediment transport from a given set of steady-state hydraulic parameters and sediment properties. Some functions compute bed-load transport, and some compute bed-material load, which is the total load minus the wash load (total transport of particles found in the bed). In sand-bed streams with high transport rates, it is common for the suspended load to be orders of magnitude higher than that found in gravel-bed or cobbled streams. It is therefore important to use a transport predictor that includes suspended sediment for such a case.

The following sediment transport functions are available in HEC-RAS:

- Ackers-White
- Engelund-Hansen
- Laursen
- Meyer-Peter Müller
- Toffaleti
- Yang

These functions were selected based on their validity and collective range of applicability. All of these functions, except for Meyer-Peter Müller, are compared extensively by Yang and Schenggan (1991) over a wide range of sediment and hydraulic conditions. Results varied, depending on the conditions applied. The Meyer-Peter Müller and the bed-load portion of the Toffaleti function were compared with each other by Amin and Murphy (1981). They concluded that Toffaleti bed-load procedure was sufficiently accurate for their test stream, whereby, Meyer-Peter Müller was not useful for sand-bed channels at or near incipient motion. The ranges of input parameters used in the development of each function are shown in Table 12-7. Where available, these ranges are taken from those presented in the SAM package user's manual (Waterways Experiment Station, 1998) and are based on the developer's stated ranges when presented in their original papers. The ranges provided for Engelund and Hansen are taken from the database (Guy, et al, 1966) primarily used in that function's development. The parameter ranges presented are not limiting, in that frequently a sediment transport function will perform well outside the listed range. For example, Engelund-Hansen was developed with flume research only, and has been historically applied successfully outside its development range. The parameter ranges are presented as a guideline only.

A short description of the development and applicability of each function follows. It is strongly recommended that a review of the respective author's initial presentation of their function be undertaken prior to its use, as well as a review of "comparison" papers such as those referenced in the preceding paragraph. References are included in Appendix A. Sample solutions for the following sediment transport methods are presented in Appendix E.

Table 12-7
Range of input values for sediment transport functions (Sam User's Manual, 1998)

Function	d	d_m	s	V	D	S	W	T
Ackers-White (*flume*)	0.04 - 7.0	NA	1.0 - 2.7	0.07 - 7.1	0.01 - 1.4	0.00006 - 0.037	0.23 - 4.0	46 - 89
Englund-Hansen (*flume*)	NA	0.19 - 0.93	NA	0.65 – 6.34	0.19 – 1.33	0.000055 – 0.019	NA	45 - 93
Laursen (*field*)	NA	0.08– 0.7	NA	0.068 – 7.8	0.67 – 54	0.0000021 – 0.0018	63 – 3640	32 - 93
Laursen (*flume*)	NA	0.011 - 29	NA	0.7 - 9.4	0.03 – 3.6	0.00025 – 0.025	0.25 – 6.6	46 - 83
Meyer-Peter Muller (*flume*)	0.4 – 29	NA	1.25 – 4.0	1.2 – 9.4	0.03 – 3.9	0.0004 – 0.02	0.5 – 6.6	NA
Tofaletti (*field*)	0.062 – 4.0	0.095 – 0.76	NA	0.7 - 7.8	0.07 – 56.7 (R)	0.000002 – 0.0011	63 - 3640	32 – 93
Tofaletti (*flume*)	0.062 – 4.0	0.45 – 0.91	NA	0.7 - 6.3	0.07 – 1.1 (R)	0.00014 – 0.019	0.8 – 8	40 - 93
Yang (*field-sand*)	0.15 – 1.7	NA	NA	0.8 - 6.4	0.04 – 50	0.000043 – 0.028	0.44 – 1750	32 - 94
Yang (*field-gravel*)	2.5 – 7.0	NA	NA	1.4 - 5.1	0.08 – 0.72	0.0012 – 0.029	0.44 – 1750	32 - 94

Where: d = Overall particle diameter, mm
d_m = Median particle diameter, mm
s = Sediment specific gravity
V = Average channel velocity, fps
D = Channel depth, ft
S = Energy gradient
W = Channel width, ft
T = Water temperature, ^{o}F
(R) = Hydraulic Radius, ft
NA = Data not available

Ackers-White: The Ackers-White transport function is a total load function developed under the assumption that fine sediment transport is best related to the turbulent fluctuations in the water column and coarse sediment transport is best related to the net grain shear with the mean velocity used as the representative variable. The transport function was developed in terms of particle size, mobility, and transport.

A dimensionless size parameter is used to distinguish between the fine, transitionary, and coarse sediment sizes. Under typical conditions, fine sediments are silts less than 0.04 mm, and coarse sediments are sands greater than 2.5 mm. Since the relationships developed by Ackers-White are applicable only to non-cohesive sands greater than 0.04 mm, only transitionary and coarse sediments apply. Original experiments were conducted with coarse grains up to 4 mm, however the applicability range was extended to 7 mm.

This function is based on over 1000 flume experiments using uniform or near-uniform sediments with flume depths up to 0.4 m. A range of bed configurations was used, including plane, rippled, and dune forms, however the equations do not apply to upper phase transport (e.g. anti-dunes) with Froude numbers in excess of 0.8.

The general transport equation for the Ackers-White function for a single grain size is represented by:

$$X = \frac{G_{gr} s d_s}{D\left(\frac{u_*}{V}\right)^n} \quad \text{and} \quad G_{gr} = C\left(\frac{F_{gr}}{A} - 1\right) \qquad \text{12-57/8}$$

Where: X = Sediment concentration, in parts per part
G_{gr} = Sediment transport parameter
s = Specific gravity of sediments
d_s = Mean particle diameter
D = Effective depth
u_* = Shear velocity
V = Average channel velocity
n = Transition exponent, depending on sediment size
C = Coefficient
F_{gr} = Sediment mobility parameter
A = Critical sediment mobility parameter

A hiding adjustment factor was developed for the Ackers-White method by Profitt and Sutherland (1983), and is included in RAS as an option. The hiding factor is an adjustment to include the effects of a masking of the fluid properties felt by smaller particles due to shielding by larger particles. This is typically a factor when the gradation has a relatively large range of particle sizes and would tend to reduce the rate of sediment transport in the smaller grade classes.

Engelund-Hansen: The Engelund-Hansen function is a total load predictor which gives adequate results for sandy rivers with substantial suspended load. It is based on flume data with sediment sizes between 0.19 and 0.93 mm. It has been extensively tested, and found to be fairly consistent with field data.

The general transport equation for the Engelund-Hansen function is represented by:

$$g_s = 0.05\gamma_s V^2 \sqrt{\frac{d_{50}}{g\left(\frac{\gamma_s}{\gamma} - 1\right)}}\left[\frac{\tau_o}{(\gamma_s - \gamma)d_{50}}\right]^{3/2} \qquad 12\text{-}59$$

Where: g_s = Unit sediment transport
γ = Unit wt of water
γ_s = Unit wt of solid particles
V = Average channel velocity
τ_o = Bed level shear stress
d_{50} = Particle size of which 50% is smaller

Laursen: The Laursen method is a total sediment load predictor, derived from a combination of qualitative analysis, original experiments, and supplementary data. Transport of sediments is primarily defined based on the hydraulic characteristics of mean channel velocity, depth of flow, energy gradient, and on the sediment characteristics of gradation and fall velocity. Contributions by Copeland (Copeland, 1989) extend the range of applicability to gravel-sized sediments. The range of applicability is 0.011 to 29 mm, median particle diameter.

The general transport equation for the Laursen (Copeland) function for a single grain size is represented by:

$$C_m = 0.01\gamma\left(\frac{d_s}{D}\right)^{7/6}\left(\frac{\tau_o^{'}}{\tau_c} - 1\right)f\left(\frac{u_*}{\omega}\right) \qquad 12\text{-}60$$

Where: C_m = Sediment discharge concentration, in weight/volume
G = Unit weight of water
d_s = Mean particle diameter
D = Effective depth of flow
$\tau_o^{'}$ = Bed shear stress due to grain resistance
τ_c = Critical bed shear stress
$f\left(\frac{u_*}{\omega}\right)$ = Function of the ratio of shear velocity to fall velocity as defined in Laursen's Figure 14 (Laursen, 1958).

Meyer-Peter Müller: The Meyer-Peter Müller bed load transport function is based primarily on experimental data and has been extensively tested and used for rivers with relatively coarse sediment. The transport rate is proportional to the difference between the mean shear stress acting on the grain and the critical shear stress.

Applicable particle sizes range from 0.4 to 29 mm with a sediment specific gravity range of 1.25 to in excess of 4.0. This method can be used for well-graded sediments and flow conditions that produce other-than-plane bed forms. The Darcy-Weisbach friction factor is used to define bed resistance. Results may be questionable near the threshold of incipient motion for sand bed channels as demonstrated by Amin and Murphy (1981).

The general transport equation for the Meyer-Peter Müller function is represented by:

$$\left(\frac{k_r}{k_r^{'}}\right)^{3/2} \gamma RS = 0.047(\gamma_s - \gamma)d_m + 0.25\left(\frac{\gamma}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma}{\gamma_s}\right)^{2/3} g_s^{2/3} \qquad \text{12-61}$$

Where: g_s = Unit sediment transport rate in weight/time/unit width
k_r = A roughness coefficient
$k_r^{'}$ = A roughness coefficient based on grains
γ = Unit weight of water
γ_s = Unit weight of the sediment
g = Acceleration of gravity
d_m = Median particle diameter
R = Hydraulic radius
S = Energy gradient

<u>Toffaleti:</u> The Toffaleti method is a modified-Einstein total load function that breaks the suspended load distribution into vertical zones, replicating two-dimensional sediment movement. Four zones are used to define the sediment distribution. They are the upper zone, the middle zone, the lower zone and the bed zone. Sediment transport is calculated independently for each zone and the summed to arrive at total sediment transport.

This method was developed using an exhaustive collection of both flume and field data. The flume experiments used sediment particles with mean diameters ranging from 0.3 to 0.93 mm, however successful applications of the Toffaleti method suggests that mean particle diameters as low as 0.095 mm are acceptable.

The general transport equations for the Toffaleti function for a single grain size is represented by:

$$g_{ssL} = M\frac{\left(\frac{R}{11.24}\right)^{1+n_v-0.756z} - \left(2d_m\right)^{1+n_v-0.756z}}{1+n_v-0.756z} \quad \text{(lower zone)} \qquad 12\text{-}62$$

$$g_{ssM} = M\frac{\left(\frac{R}{11.24}\right)^{0.244z}\left[\left(\frac{R}{2.5}\right)^{1+n_v-z} - \left(\frac{R}{11.24}\right)^{1+n_v-z}\right]}{1+n_v-z} \quad \text{(middle zone)} \qquad 12\text{-}63$$

$$g_{ssU} = M\frac{\left(\frac{R}{11.24}\right)^{0.244z}\left(\frac{R}{2.5}\right)^{0.5z}\left[R^{1+n_v-1.5z} - \left(\frac{R}{2.5}\right)^{1+n_v-1.5z}\right]}{1+n_v-1.5z} \quad \text{(upper zone)} \qquad 12\text{-}64$$

$$g_{sb} = M\left(2d_m\right)^{1+n_v-0.756z} \quad \text{(bed zone)} \qquad 12\text{-}65$$

$$M = 43.2C_L\left(1+n_v\right)VR^{0.756z-n_v} \qquad 12\text{-}66$$

$$g_s = g_{ssL} + g_{ssM} + g_{ssU} + g_{sb} \qquad 12\text{-}67$$

Where: g_{ssL} = Suspended sediment transport in the lower zone, in tons/day/ft
g_{ssM} = Suspended sediment transport in the middle zone, in tons/day/ft
g_{ssU} = Suspended sediment transport in the upper zone, in tons/day/ft
g_{sb} = Bed load sediment transport in tons/day/ft
g_s = Total sediment transport in tons/day/ft
M = Sediment concentration parameter
C_L = Sediment concentration in the lower zone
R = Hydraulic radius
d_m = Median particle diameter
z = Exponent describing the relationship between the sediment and hydraulic characteristics
n_v = Temperature exponent

Yang: Yang's method (1973) is developed under the premise that unit stream power is the dominant factor in the determination of total sediment concentration. The research is supported by data obtained in both flume experiments and field data under a wide range conditions found in alluvial channels. Principally, the sediment size range is between 0.062 and 7.0 mm with total sediment concentration ranging from 10 ppm to 585,000 ppm. Channel widths range from 0.44 to1746 ft, depths from 0.037 to 49.4 ft, water temperature from 0^o to 34.3^o Celsius, average channel velocity from 0.75 to 6.45 fps, and slopes from 0.000043 to 0.029.

Yang (1984) expanded the applicability of his function to include gravel-sized sediments. The general transport equations for sand and gravel using the Yang function for a single grain size is represented by:

$$\log C_t = 5.435 - 0.286\log\frac{\omega d_m}{\nu} - 0.457\log\frac{u_*}{\omega} + \left(1.799 - 0.409\log\frac{\omega d_m}{\nu} - 0.314\log\frac{u_*}{\omega}\right)\log\left(\frac{VS}{\omega} - \frac{V_{cr}S}{\omega}\right)$$

$$\text{for sand } d_m < 2mm \qquad \text{12-68}$$

$$\log C_t = 6.681 - 0.633\log\frac{\omega d_m}{\nu} - 4.816\log\frac{u_*}{\omega} + \left(2.784 - 0.305\log\frac{\omega d_m}{\nu} - 0.282\log\frac{u_*}{\omega}\right)\log\left(\frac{VS}{\omega} - \frac{V_{cr}S}{\omega}\right)$$

$$\text{for gravel } d_m \geq 2mm \qquad \text{12-69}$$

Where: C_t = Total sediment concentration
ω = Particle fall velocity
d_m = Median particle diameter
ν = Kinematic viscosity
u^* = Shear velocity
V = Average channel velocity
S = Energy gradient